STORMWATER TREATMENT AREA NO. 3 & 4

PLAN FORMULATION DOCUMENT

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4.1 HYDROGEOLOGIC FIELD INVESTIGATIONS

A Hydrogeologic Field Investigation was performed at the South Florida Water Management District (SFWMD) Stormwater Treatment Area No. 3 and 4 (STA 3/4) to collect site specific data for use in seepage and groundwater modeling evaluations. The references in this text referring to the internal configuration of the STA and the cell designations reflect the configuration defined in the General Design Memorandum, and may be inconsistent with the cell numbering for the final design. The site, shown in **Figure 4.1**, is located on the west side of State Road 27, approximately 25 miles south of South Bay, Florida. The SFWMD plans to develop STA 3/4 to treat agricultural runoff and discharge from the Everglades Agricultural Area (EAA) conveyed via a series of canals.

This investigation included the installation of five 4-inch diameter test wells to the base of the water producing intervals of the surficial aquifer system to define the hydrogeology of the site. Following testing and analysis of the five wells, six 2-inch diameter monitor wells were installed in an array around one of the test wells. A 96-hour aquifer performance test (APT) was conducted in one of the five test wells to develop estimated hydrogeologic parameters for use in the computer modeling evaluations.

The project also included the development of a comprehensive field plan to detail drilling and testing techniques and document activities that would occur during the investigation. The field plan (previously provided to the SFWMD) is reproduced in **Appendix A**.

4.1.1 Background

A review of previous investigations conducted at STA 3/4 was performed to collect existing site subsurface information such as lithologic, hydrologic, and geophysical data. Soil survey maps and other STA site data were reviewed, and applicable information extrapolated. In addition, a number of hazardous waste investigations have been performed in the STA 3/4 area, however little information was available on the lithology and hydrology at depths greater than 15 feet below land surface (bls). The reference section contains a listing of the reports used in this investigation.

4.1.2 Well Construction And Testing

A series of five 4-inch diameter test wells were drilled to determine lithologic and hydrologic characteristics at STA 3/4. The locations of the test wells are shown in **Figure 4.1**. In addition to these wells, a comprehensive geotechnical investigation was undertaken at STA 3/4 to better define the lithologic layering and obtain the engineering characteristics of the surficial sediments. The test wells were drilled at locations identified for the geotechnical investigation; therefore the location identification numbers were included with the test well identification numbers for later correlation. At the completion of well construction, an 8-hour pump test was performed on each well to estimate hydrogeologic characteristics of each test well site. A summary of the results of the pump tests is shown in **Table 4.1**.

4.1.3 Test Well No. 1 (Location No. 53)

Construction of Test Well No. 1 (TW-1) at Location No. 53 (**Figure 4.1**) began on April 28, 1999. A 6-inch diameter pilot hole, drilled by the mud rotary method, was advanced to 120 feet below land surface (bls). Geophysical logging was performed following completion of the pilot borehole. Geophysical logging consisted of a caliper, long and short normal electric, gamma ray, and spontaneous potential logs. The pilot hole subsequently was reamed to a nominal 10-inch diameter to a depth of 65 feet bls. The 4-inch diameter Schedule 40 PVC casing was set to 35 feet with 30 feet of 0.020-slot “Vee-Wire” PVC screen set from 35 to 65 feet bls. Gravel was placed into the annular space around the screen by tremie pipe to 33 feet bls and a 2-foot sand cap was placed above the gravel. The well was cemented to surface using 4-percent gel cement. Construction of Test Well No. 1 was completed on May 3, 1999.

Insert Table 4.1

A generalized test well completion diagram is shown in **Figure 4.2**. Aside from specific depths (included in the table), **Figure 4.2** is representative of the completion method used for all five test wells. Geophysical logs and associated lithologies for all test wells can be found in **Appendix B**. Also included in **Appendix B** is a graphic comparing lithology and the geophysical logs for each test well. This graphic shows the lithology, screened interval, drilling characteristics, and all of the geophysical logs performed in a one-page format.

Air-lift development was performed on May 5, 1999 for approximately 4.5 hours. Additional air-lift development was conducted on June 1, 1999 for 2 hours. Following air-lift development a surface pump was installed in TW-1 and the well was pumped at rates ranging from 100 to 190 gallons per minute (gpm) for 3 hours. Development progressed until the well discharge was free of sand and specific capacity stabilized.

On June 2, 1999, a 4-hour step-rate pumping test was performed on TW-1 at 55 gpm, 98 gpm, 140 gpm, and 186 gpm. Drawdown was monitored in TW-1 by a pressure transducer/data logger and supplemented with intermittent hand-level readings. On June 3, 1999, an 8-hour constant-rate test was performed. The well had 4.40 feet of drawdown at 186 gpm prior to a heavy rain event. Results of the step- and constant-rate pumping tests are included in **Appendix C**.

4.1.4 Test Well No. 2 (Location No. 59)

Construction of Test Well No. 2 (TW-2) at Location No. 59 (**Figure 4.1**) began on May 4, 1999. A 6-inch pilot hole was drilled to 120 feet bls. The pilot hole was geophysically logged on May 5, 1999. Based on cuttings, drilling characteristics, and geophysical logs, the interval from 30 to 60 feet bls was selected for screening. The pilot hole was reamed to a nominal 10-inch diameter to 60 feet bls, and the 4-inch PVC "Vee-Wire" 0.020-slot screen and PVC casing were installed. The gravel pack was washed in place by tremie to 3 feet above the screen. A 5-foot sand cap was placed over the gravel, and the well was cemented to surface with 4-percent gel cement.

The well was developed by the airlift method on May 7, 1999 for 4 hours. Additional air-lift development was conducted on June 2, 1999 for 5 hours and June 3, 1999 for 1 hour. Following air-lift development a surface pump was installed in TW-2 and the well was pumped at rates ranging from 60 to 192 gpm for approximately 9 hours. Development progressed until the well discharge was free of sand and specific capacity stabilized.

On June 6, 1999, a 4-hour step-rate pumping test was performed on TW-2 at 67 gpm, 105 gpm, 151 gpm, and 190 gpm. Drawdown was monitored in TW-2 by both a pressure transducer/data logger and intermittent hand-level readings. On June 7, 1999, an 8-hour constant-rate test was performed at 190 gpm. The well had a drawdown of 13.2 feet prior to a heavy rain event. Results of the step- and constant-rate pumping tests are included in **Appendix C**.

4.1.5 Test Well No. 3 (Location No. 62)

Construction of Test Well No. 3 (TW-3) at Location No. 62 (**Figure 4.1**) began on May 10, 1999. A 6-inch pilot hole was drilled to 120 feet bls. The pilot hole was geophysically logged on May 11, 1999. Based on cuttings, drilling characteristics, and geophysical logs, the interval from 50 to 80 feet bls was selected for screening. The pilot hole was reamed to a nominal 10-inch diameter to 81 feet bls, and the 4-inch PVC “Vee-Wire” 0.020-slot screen and PVC casing were installed. The gravel pack was washed in place by tremie to 5 feet above the screen. A 5-foot sand cap was placed over the gravel, and the well was cemented to surface with 4-percent gel cement.

The well was developed by the airlift method on May 13, 1999 for 7 hours. Additional air-lift development was conducted on June 3, 1999 for approximately 3 hours. Following air-lift development a surface pump was installed in TW-3 and the well was pumped at rates ranging from 47 to 210 gpm for approximately 6.5 hours. Development progressed until the well discharge was free of sand and specific capacity stabilized.

On June 9, 1999, a 4-hour step-rate pumping test was performed on TW-3 at 56 gpm, 110 gpm, 147 gpm, and 210 gpm. Both a pressure transducer/data logger and intermittent hand-level readings monitored drawdown in TW-3. On June 10, 1999, a 7-hour constant-rate test was performed at 210 gpm. During the test, a discharge coupling developed stress fractures. Discharge rates began to increase as the leak became worse. The pump test was terminated after 7 hours of pumping to insure collection of recovery data prior to rupturing the discharge line. The well had a drawdown of 4.26 feet at 210 gpm. Results of the step- and constant-rate pumping tests are included in **Appendix C**.

4.1.6 Test Well No. 4 (Location No. 68)

Construction of Test Well No. 4 (TW-4) at Location No. 68 (**Figure 4.1**) began on May 13, 1999. A 6-inch pilot hole was drilled to 120 feet bls. The pilot hole was geophysically logged on May 17, 1999. Based on cuttings, drilling characteristics, and geophysical logs, the interval from 40 to 70 feet bls was selected for screening. The pilot hole was reamed to a nominal 10-inch diameter to 70 feet bls, and the 4-inch PVC "Vee-Wire" 0.020-slot screen and PVC casing were installed. The gravel pack was washed in place by tremie to 5 feet above the screen. A 2-foot sand cap was placed over the gravel, and the well was cemented to surface with 4-percent gel cement.

The well was developed by the air-lift method on May 19, 1999 for approximately 4 hours. Additional air-lift development was performed on June 3, 1999 for approximately 2 hours. A surface pump was installed in TW-4 on June 14, 1999. The well was pump developed at rates ranging from 46 to 154 gpm for approximately 5.5 hours. Development progressed until the well discharge was free of sand and specific capacity stabilized.

On June 14, 1999, a 4-hour step-rate pumping test was performed on TW-4 at 53 gpm, 80 gpm, 122 gpm, and 156 gpm. Both a pressure transducer/data logger and intermittent hand-level readings monitored drawdown in TW-4. On June 15, 1999, an 8-hour constant-rate test was performed at 154 gpm with a drawdown of 15.88 feet. Results of the step- and constant-rate pumping tests are included in **Appendix C**.

4.1.7 Test Well No. 5 (Location No. 75)

Construction of Test Well No. 5 (TW-5) at Location No. 75 (**Figure 4.1**) began on May 18, 1999. A 6-inch pilot hole was drilled to 120 feet bls. The pilot hole was geophysically logged on May 19, 1999. Based on cuttings, drilling characteristics, and geophysical logs, the interval from 55 to 85 feet bls was selected for screening. The pilot hole was reamed to a nominal 10-inch diameter to 85 feet bls, and the 4-inch PVC "Vee-Wire" 0.020-slot screen and PVC casing were installed. The gravel pack was washed in place by tremie to approximately 20 feet above the screen. A 5-foot sand cap was placed over the gravel, and the well was cemented to surface with 4-percent gel cement.

On May 21, 1999,the well was developed by the airlift method for approximately 4 hours. Additional air-lift development was performed between June 3, 1999, and June 10, 1999 for approximately 60 hours. The extended development time likely was caused by bridging of the gravel-pack during installation. In addition to air development, approximately 4 gallons of commercial bleach (4% sodium hypochlorite) was used to break up any drilling mud in the screen. On June 10, 1999, TW-5 was recovering immediately following surging and was producing clean water. A surface pump was installed in TW-5 on June 16, 1999. The well was pump developed at rates ranging from 123 to 210 gpm for approximately 5 hours. Development progressed until the well discharge was free of sand and specific capacity stabilized.

On June 16, 1999, a 4-hour step-rate pumping test was performed on TW-5 at 70 gpm, 111 gpm, 156 gpm, and 209 gpm. Drawdown was monitored in TW-5 by both a pressure transducer/data logger and intermittent hand-level readings. On June 17, 1999, an 8-hour constant-rate test was performed at 210 gpm with a drawdown of 6.69 feet. Results of the step- and constant-rate pumping tests are included in **Appendix C**.

4.1.8 Regional Geology/Hydrogeology

The following section describes the regional geology in the vicinity of STA 3/4 described in published reports from the area. Actual geologic data collected during well installation has been correlated to the published lithologic units and is presented in **Appendix B**.

The Soil Survey of Palm Beach County (McCollum et al., 1978) classifies the soils in and around STA 3/4 as the Pahokee Muck. These soils range in thickness from 3 to 5 feet and are classified as nearly level, very poorly drained organic soils (muck). The muck is described as containing an upper 2 to 2.5 feet of black muck with 1 to 2 feet of dark reddish brown muck extending to the limestone below. Its permeability is described as rapid. Numerous small areas of Lauderhill, Terra Ceia, Torry, and Okeelanta soils are also common. These soils are more fibrous in texture, are less decomposed, and contain a thin sandy layer at their base above the limestone.

Below the surficial organics and mucks, the geology of the region consists of calcareous muds and sands, and limestones of the Lake Flirt Marl, the Fort Thompson Formation, the Caloosahatchee Formation, and the upper Tamiami Formation (Scott, 1992; Miller, 1988; Parker et al., 1955; and Schroeder et al., 1954). These units comprise the surficial aquifer system in the region to a depth of approximately 120 feet bls (Land et al., 1973). Based on data from the installation of an injection well within the EAA, CH2MHill (1990) inferred the depth of the surficial aquifer as approximately 200 feet deep.

The Lake Flirt Marl is principally a light-gray, fresh-water, calcareous mud deposit that usually lies in direct contact with the surficial rocks of the underlying Fort Thompson Formation. Thickness of the Lake Flirt Marl ranges from 0 to 6 feet, and where present, is relatively impermeable acting as a semi-confining unit to the underlying more permeable Fort Thompson.

The Fort Thompson Formation consists of interbedded layers of shell, calcareous marine sands (calcilutite), and limestone (Johnson, 1989). The sands are moderate brown to white, fine to medium grained, moderately rounded, with intergranular porosity. Pebble to cobble size limestones are present in the sand and shell layers. The limestones are light brown in color, microcrystalline and skeletal with moldic porosity. L.J. Nodarse and Associates, Inc. (1996a, b) described the caprock and underlying, more weathered limestone at STA 5 and STA 6 as being representative of the Fort Thompson Formation. They reported average calculated mean permeability rates for the weathered zones at STA 6 of 1.4 to 15 feet per day (ft/day) at depths from 4 to 9 feet bls.

The Caloosahatchee Formation is described approximately the same as the Fort Thompson Formation with the addition of finer silts and clays. Below the Caloosahatchee Formation lies the Tamiami Formation. It varies from creamy white limestone to a greenish-gray clay and marl. The permeability is moderate to low.

Below the Tamiami Formation is the impermeable sandy clay of the Hawthorn Group. The lower Tamiami Formation and the Hawthorn Group form the confining unit between the surficial aquifer and the Floridan Aquifer System (Scott, 1992). The upper Floridan aquifer system in this area is composed of the Suwannee and Ocala Limestones, and the Avon Park Formation.

Brown and Caldwell (1996) regionally divided the limestone in the EAA, including STA 2, into upper and lower zones based on permeability differences. The upper zone is described as a locally discontinuous seam of low permeability caprock followed by a deep limestone formation with moderate to extremely high permeability. The upper zone was assigned vertical and horizontal permeabilities of 40 ft/d and 200 ft/d, respectively, while the lower zone was thought to be 250 percent more permeable.

In general, the 5 test wells drilled at STA 3/4 were similar with respect to geology. The uppermost 1 to 2 feet consisted of a rich, highly organic dark brown muck. Underlying the muck was a hard “caprock” consisting of an orange to brown, well indurated limestone mixed with varying percentages of shell and sand ranging in thickness from 5 to 9 feet. At TW-5, between 1 and 2 feet bls, was a layer of gray sandy clay and/or silt.

This layer lies between the muck and limestone. Underlying the caprock, is a more poorly indurated gray to white limestone with variable amounts of the harder tan to brown limestone, shell, and sand. A second well-indurated zone was encountered in TW-1, TW-4, and TW-5 between 50 and 70 feet bls. A silty sand was encountered in 4 of the 5 test wells at depths ranging from 75 to 90 feet bls. A summary of the lithologies by depth for each of the test wells is provided in **Appendix B**. The summary sheets also contain an indicator showing the relative "hardness", or drilling characteristics encountered during construction of the wells. In addition, **Appendix B** contains cross-sections depicting the lithology with respect to the geophysical log correlation for each well.

The above descriptions generally agree with the findings from previous investigations at the STA 3/4 area and at STA 1, STA 2, STA 5, and STA 6. Subtle variations in thickness and composition are common between the different sites. Based on the lithologies encountered, the zone with the highest probability of water production was between the caprock and the siltier sediments. This zone primarily consisted of variable quantities of limestone, shell, and sand, indicative of the Fort Thompson Formation.

4.1.9 Geophysical Log Interpretation

Geophysical logs were performed in the pilot borehole of each test well prior to reaming and setting casing. The logging suite performed included a caliper log, gamma ray log, spontaneous potential log, and long and short normal electric log.

The caliper log measured the pilot borehole diameter and gave an indication of the hardness and degree of lithification of the formation material. Test Wells No. 4 and 5 have nearly gauge pilot boreholes with diameters ranging from 4 to 9 inches. A few small wash-out areas occurred in each of these wells. The wash-outs corresponded to the portions of the caliper log that showed a larger diameter. Test Wells No. 1, 2, and 3 display nearly gauge boreholes in the top 30 feet and bottom 50 feet of each well. In the intermediate 40 feet of each borehole the caliper log showed wash-outs greater than 10 inches in diameter. This indicated that the material in this interval was softer and less lithified than in the upper and lower portions of the borehole

The gamma ray log is used to measure the natural radioactivity of the materials through which the borehole passes. The natural radioactivity is highlighted on this log by peaks on the graph, indicating the presence of either clay of phosphate. Based on comparison of this log to the lithology encountered during drilling, layers of reduced permeability (potential semi-confining units) can be identified. Where present, this reduced permeability would impede seepage losses from within the STA. The gamma ray logs show two layers of high gamma ray activity. The first layer of high gamma ray activity was found at approximately 15 feet bls and extended to a depth of approximately 30 feet bls. The second layer of high gamma ray activity begins at approximately 50 feet bls and extends to depths between 70 and 90 feet bls. Both of these layers were found at shallower depths in the western wells and also thin towards the west.

Two electric logs were performed in each of the test wells, the long and short normal resistivity (LSN) log and the spontaneous potential (SP) log. These logs give an indication of zones of greater permeability which might result in greater seepage losses fom the STA. The LSN resistivity log was used to indicate the presence or absence of water producing intervals. The water producing interval began at approximately 25 to 30 feet bls and extends to approximately 75 feet bls. Below approximately 75 feet bls the long and short normal resistivity curves overlay indicating the bottom of the highly productive interval.

The spontaneous potential (SP) log gives an indication of water quality changes with depth. The SP logs show a negative deflection at approximately 25 feet bls indicating the top of the water producing interval. Below 25 feet bls the SP logs are relatively flat and featureless although, a slight increase in salinity is observed with increasing depth.

These four logs, in combination with the drill cuttings, were used to select the screened interval for the test wells. The screened interval was chosen where the electric log indicates a water-bearing interval, and where the caliper log shows a slight wash-out indicating the ability to deliver water.

4.1.10 Screened Interval Selection

At the completion of each pilot hole and geophysical logging suite, data was compiled and discussed by Montgomery Watson personnel to determine the most suitable aquifer interval to screen. The zones were selected based on lithology, drilling characteristics, and geophysical logging results. In all cases, the deepest zone still capable of yielding a high volume of water was selected.

4.1.11 Test Well Analysis

The data obtained during the step- and constant-rate pumping test were analyzed to determine the transmissivity and hydraulic conductivity for the aquifer system at each of the test wells. Each well was analyzed using the Bierschenk (1964) Method to determine individual well efficiencies. Decreases in well efficiency are attribute to poor specific capacity and may be caused by insufficient development, improper well construction, damage to the formation during drilling, or poor filter-pack design. Well efficiency is determined using the difference between the drawdown outside the casing and the pumping level inside the casing (Driscoll, 1986). The water level data from the constant-rate pumping tests was then analyzed using the Theis Recovery, Johnson, and Mace Methods. Results of these analyses are contained in **Appendix C**. A summary of the results is presented as **Table 4.1**.

4.1.12 APT Test Well Selection

A meeting was held on June 23, 1999, at the Montgomery Watson office, to discuss the results of the work to date and determine which of the test wells would be used for a 96-hour APT. Attending the meeting were Randy Bushey (SFWMD), Mark Abbott (MW), Anne Murray (MW), and Neil Johnson (MW). During the meeting, the following topics were addressed:

- Summary of field activities
- Discussion of local geology
- Results of the short-term pumping tests
- Discussion of field constraints
- Evaluation of data
- Selection of site
- Monitor well spacing
- Monitor well construction and screened intervals
- Pump test schedule

Based on the information reviewed, TW-3 was selected as the most suitable location for the APT. TW-3 had the second highest well efficiency (41%), and the highest specific capacity (48 gallons per minute per foot) of the 5 wells. The transmissivity at TW-3 ranged from 95,455 to 224,928 gallons per day per foot (gpd/ft) depending on the analysis method. In addition, TW-3 was screened from 50 to 80 feet bls in a zone intermediate to the other wells. TW-3 also was one of the most suitable wells for the placement of the monitor well array. The site was not highly vegetated and only one drainage canal interfered with access to the monitor wells.

The monitor wells were configured near TW-3, as shown in **Figure 4.3**. The four deeper monitor wells were screened in the same interval as TW-3 (50 to 80 feet bls). The two shallow monitor wells were screened from 10 to 15 feet bls. Distances selected for the monitor wells were based on Theis distance/drawdown curves as illustrated in **Figure 4.4**. A generalized well completion diagram for the six monitor wells is provided in **Figure 4.5**. Three staff gauges were installed in the canals adjacent to TW-3. The staff gauge locations are identified in **Figure 4.3**. **Table 4.2** summarizes the monitor well locations and screened intervals.

Table 4.2

APT Test/Monitor Well and Staff Gauge Location Summary

Well/Staff Gauge ID	Top of Casing or Reference Elev. (feet NGVD)	Radial Distance from TW-3 (feet)	Screened Interval (feet bls)
TW-3	11.79	-	50 - 80
MW-1D	14.22	910.7	50 - 80
MW-2D	12.89	389.2	50 - 80
MW-2S	13.05	390.9	10 – 15
MW-3D	12.37	48.9	50 – 80
MW-3S	12.11	51.0	10 - 15
MW-4D	12.61	406.1	50 - 80
Staff Gauge #1	+2.13	1390.3	-
Staff Gauge #2	+7.91	1513.4	-
Staff Gauge #3	-3.19	1541.8	-

On June 28, 1999, the monitor well sites were field-located using a compass and measuring wheel. Drilling of the first monitor well, MW-1D, began on June 29, 1999. Monitor well drilling was completed on July 4, 1999. All monitor wells were air-lift developed upon completion until the water was clear of silt and sand. The 3 staff gauges were installed prior to starting the APT on July 19, 1999.

4.2 AQUIFER PERFORMANCE TESTING

To determine the aquifer parameters of the subsurface material at the site, a 96-hour APT was performed. The 96-hour duration was selected in order to determine leakance effects. One pumping well (TW-3) and 6 monitoring wells, as listed above, were used for this test. The distances between the pumping well and the monitor wells were determined using the Theis distance drawdown method as discussed previously. The maximum radial distance of 1,000 feet was selected where a minimum drawdown of between 0.2 and 0.4 feet would occur based on transmissivities ranging from 100,000 to 200,000 gpd/ft and discharge between 160 and 210 gpm.

The area around STA 3/4 had experienced several weeks of rain and the site was still inundated with standing water. On July 12, 1999 the SFWMD constructed an earthen dam in the canal south of TW-3, isolating approximately 2/3 of the site in an effort to drain the flooded areas,. The Mace Pump Station was started to assist in dewatering the area around TW-3 by creating a gradient towards the east/west canal to the south of the site. That same day, pressure transducers connected to electronic data loggers were placed in TW-3 and in the 6 monitor wells to begin collecting background water level data. A rain gauge and barometer were installed adjacent to MW-2S to collect atmospheric data during the APT.

On July 16, 1999, a site visit revealed that the standing water on the site had been removed and the water level in the canal east of the dam had dropped by approximately 6 feet. The drilling contractor re-mobilized on July 19, 1999, and prepared to commence the APT, which began at 9:55 AM on July 20, 1999.

During an inspection of the pump by SFWMD staff on the morning of July 20, 1999, the Mace Pump Station was shut off due to cavitation. On July 21, 1999, during routine monitoring of the wells and staff gauges, it was determined that the water levels in five of the monitor wells had risen above the static water level recorded before the test. The cause of the increased water level was linked to the cessation of pumpage from the Mace Pump Station and the subsequant rise on canal levels as runoff and groundwater discharge continued. Due to the increase in water levels, the APT pump was shut down. The APT was postponed until the canal returned to its static level.

On August 2, 1999, a second APT was started at 10:54 AM. On August 3, 1999, the discharge coupling, adjacent to the pump, ruptured. The pump was shut down and water levels were allowed to recover (for 24 hours)

The third APT began on August 4, 1999, at 13:10 hrs. Discharge during the pumping test was maintained at approximately 200 gpm. Discharge was measured using a 3-inch totalizing flowmeter installed in the discharge line. The discharge line ran approximately

1,300 feet south of TW-3 across the southern canal. In addition to the data loggers, hand level measurements were taken rapidly in all seven wells during the initial pumping period. The hand water level measurements provided calibration points for the data loggers and back-up data in case of a malfunction. Additionally, the three staff gauges and an existing staff gauge in the southern canal, located approximately 1.5 miles east of TW-3, were monitored during the APT.

Several rainfall events occurred during the APT. On August 5, 1999, at 09:30 hrs, the rain gauge indicated that 0.09 inches of rain had fallen since the start of the test. On August 6, 1999, at 09:46 hrs, the rain gauge showed 0.03 inches. On the afternoon of August 6, 1999, between 16:00 hrs and 16:30 hrs, the site received 0.34 inches of rain. This rain event was detected in all of the wells as an instantaneous increase in water level of approximately 0.14 feet. Between August 7, and August 8, 1999, an additional 0.04 inches of rain fell on the site. No rainfall was recorded between August 8, and August 9, 1999.

Discharge during the APT was maintained at approximately 200 gpm. Rates were monitored using 2 separate 3-inch totalizing flowmeters. The flowmeters began clogging on August 6, 1999. It appeared that a fine, lime silt was clogging the impeller on the flowmeter. After one flowmeter would start to clog, the discharge water was diverted via a bypass, and the flowmeters were exchanged.

At 13:11, August 8, 1999, the pump was shut down and recovery of the groundwater levels was monitored. Hand water level measurements were taken rapidly in all 7 wells during the initial recovery period similar to the initial pumping period. Recording of recovery was stopped on August 9, 1999 and the wells and staff gauges were surveyed for location and elevation.

4.2.1 APT DATA ANALYSIS

The APT test data was analyzed using methods developed by Theis, Walton, Neuman, Thiem, and Jacob (Kruseman and de Ridder, 1989). Each of these methods assumes that the aquifer has a seemingly infinite areal extent; is homogeneous, isotropic, and of uniform thickness; is pumped at a constant discharge rate; and the pumping well fully penetrates the aquifer resulting in horizontal flow.

The Theis, Walton, and Neuman methods are used when the aquifer is in an unsteady state. These methods are graphical curve-fitting methods relating the drawdowns observed in each of the monitoring wells over time to different type curves. The Theis method was developed for unsteady-state flow in a confined aquifer. Walton's method incorporates an additional component for leaky aquifers. Neuman's method, designed for pumping test in unconfined aquifers, addresses the concept of delayed water table response. In unconfined aquifers, the water levels in observation wells near the pumping well tend to decline at a slower rate than confined aquifers. As an unconfined aquifer is pumped, there is a rapid decline in the water table adjacent to the pumping well. As pumping continues, the cone of depression expands and deepens. This phenomenon is responsible for the "s-shaped curve" graphs typical for unconfined aquifers. The Neuman method also provides a component for determining vertical hydraulic conductivity.

Thiem's method assumes that the aquifer has reached steady state. Transmissivity is determined from monitor well pairs (some radial distance from the pumping well) and the difference in the observed drawdowns. Jacob's constant time method relates the amount of drawdown observed in each monitor well, at various offset distances from the pumping well, at an instantaneous moment in time.

One of the major assumptions for each of the analytical methods is that the pumping well fully penetrates the aquifer. For this APT, the pumping well is screened in only the lower portion of the aquifer. The partial penetration of the pumping well induces vertical flow patterns within the aquifer. This causes the flow velocities adjacent to the well to be greater than they would be otherwise, leading to increased head loss. This effect is strongest adjacent to the well and decreases with increasing distance from the well. The effect is negligible at a distance of 1.5 to 2 times greater than the saturated thickness of the aquifer (Kruseman and de Ridder, 1990).

Therefore, the APT data can be grouped into two categories: the wells monitoring the interval screened by the pumping well; and the wells monitoring intervals shallower than that screened by the pumping well. Because of the vertical flow components, the monitoring wells that have small offset distances were eliminated from the analysis. Furthermore, the Thiem and Jacob methods were analyzed for wells completed/screened in the same depth interval.

Additional data collected during the APT, rainfall and barometric pressure, were used to aid in the determination of the drawdown. Each rainfall event was observed simultaneously in each of the monitoring wells indicating that the aquifer was connected from ground surface to the screened interval. The response to each rainfall event is seen as increases in water level. Barometric pressure changes only affect confined aquifers and had no effect on the drawdown data.

During the course of the APT, the canal south of TW-3 rose approximately 0.12 feet. This rise in local water level was also observed in the test and observation wells. In addition, a rainfall event of 0.34 inches occurred on August 4, 1999 at 1600 hrs. This event caused an immediate increase in the water table of 0.15 feet and in the southern canal of 0.12 feet. The canal west of the dam rose approximately 0.10 feet and the canal to the north by 0.08 feet. The obliquity of the increase in water level over the site is evident by the variation in increases in background water level from west to east. Variation in background was also evident from north to south. Monitor well MW-4D recovered to 0.10 feet above background while TW-3 (400 feet south) recovered to 0.12 feet above background.

Data from all of the wells was corrected for the increase in static water level by adding a correction factor of 0.03 to 0.13 feet over the test period. From the start of pumping, data points at 15-minute intervals were normalized over 120 hours to correct for the water level rise. This correction does not account for the large rain event. The corrected data was used in the Theim and Jacob methods. The uncorrected data was utilized for the Theis, Walton, and Neuman methods where slight variations in drawdown will not have an effect plotted on a logarithmic scale.

Finally, partial penetration effects were investigated for TW-3, MW-3D, and MW-3S. The vertical component of flow associated with partial penetration is insignificant in wells two or more aquifer thicknesses away. Kozeny (1933) provides a method for estimating results from partially penetrating wells in confined aquifers that are relatively homogeneous. The equation is provided in **Appendix D**. The specific capacity of TW-3 increased 195% from 51.95 gpm/ft to 101.2 gpm/ft. This correction would increase the discharge in calculations for TW-3 basically doubling the transmissivity.

Since precise modeling using computations describing natural systems is unrealistic and an APT pumping rate can not be maintained at an exactly constant rate for the entire pumping period, the various calculations from each analysis method represent approximations. Each method has advantages and limitations, so it is typical to select a number of analytical methods and achieve a reliable average value. **Table 4.3** below summarizes the results of each method of pump test analysis. A detailed mathematical evaluation and graphical presentations of each analysis method can be found in **Appendix D**.

Table 4.3

Pump Test Analysis Results

Method	**Transmissivity (gpd/ft)**						
	TW-3	**MW-1D**	**MW-2D**	**MW-2S**	**MW-3D**	**MW-3S**	**MW-4D**
Theis Curve	5,093	881,538	254,667	1,763,077	69,455	1,528,000	305,600
Walton Curve	2,491	694,545	191,000	1,910,000	54,571	996,522	254,667
Neuman Curve	-	342,166	191,000	658,416	-	654,857	244,827
Thiem Method	74,547	-	151,627	-	204,601	-	345,900
Jacob Constant-Time	Deep Wells 224,681						

The results of the APT data analysis for the observation wells located 400 feet from the pumped well (MW-2D and MW-4D) indicate that the transmissivity for the interval screened by the pumping well averages 198,000 gallons per day per foot (gpd/ft). This value is reasonably consistent with those published in the literature for the nearby EAA and other STA's which ranged from 22,000 to 749,000 with an average transmissivity of 436,538 gpd/ft at similar depths.

4.3 TWO-DIMENSIONAL SEEPAGE MODELING

A seepage analysis was performed to estimate the steady-state seepage rates and ground water levels across selected vertical cross sections of the proposed STA 3/4 impoundment. Three water level scenarios were analyzed for each of the cross sections developed for the STA.

4.3.1 Seepage Analysis Methodology

For this analysis, the SEEP2D model was selected because of its ability to estimate seepage flows and groundwater levels in a vertical cross section. SEEP2D is a steady-state, finite element, two-dimensional model developed by the U.S. Army Corps of Engineers. Because it incorporates canal cross sections into the model grid, it is well suited to calculate seepage flows across the interface between the groundwater and surface water system.

4.3.2 Cross Section Description

To estimate seepage losses, seven cross sections have been developed for the STA (**Figure 4.6**). The cross sections include two east-west cross sections bisecting the eastern perimeter levee, two east-west cross sections bisecting the western perimeter levees, one north-south cross section that bisects the northern perimeter levee, one north-south cross section that bisects the southern perimeter levee, and one east-west cross section that bisects the supply canal.

Two cross sections were developed along the eastern perimeter of the STA because of the variability of the subsurface materials found. One cross section was located in the northern portion and one was located to the south. The cross sections cover approximately 2,200 feet. They originate 1,000 feet west of the eastern perimeter levee in the STA, cross the levee and seepage collection system, bisect Highway 27 and the North New River Canal, and continue 1,000 feet east into the adjoining farm lands.

The north-south cross section bisecting the northern perimeter levee system covers approximately 2,450 feet. It originates 1,000 feet south of the levee and collection system within the STA, crosses the distribution canal, inflow levee, inflow canal, exterior levee, and seepage collection canal, and continues 1,000 feet north into the adjoining farm lands.

The supply canal cross section covers approximately 2,500 feet. It begins 1,000 feet west of the existing Holey Lands levee borrow canal, crosses the borrow canal, Holey Lands levee, supply canal, exterior levee, and seepage collection canal, then extends 1,000 feet east into the adjoining farm lands.

The three remaining cross sections are located in the western portion of the STA. The western perimeter of the STA is offset by an approximate 3 mile by 2 mile (6 square mile) cut-out area called the "Toe of the Boot". The western most cross section is located north of the "Toe of the Boot" and covers approximately 2,100 feet. It originates 1,000 feet west of the Holey Lands borrow canal, continues east across the borrow canal and Holey Lands levee, and continues an additional 1,000 feet into the STA. The second east-west cross section is located on the southern portion of the western perimeter levee in the "Toe of the Boot". It originates 1,000 feet west of the Holey Lands levee in the "Toe of the Boot", continues east across the Holey Lands levee, the outfall canal, and west perimeter levee, and continues an additional 1,000 feet into the STA. The north-south cross section bisecting the southern perimeter levee system covers approximately 2,150 feet. It originates 1,000 feet north of the Cell 5 collection canal within the STA, continues south across the collection canal and Holey Land levee, and extends 1,000 feet further into the Holey Lands in the "Toe of the Boot".

4.3.3 Layering and Horizontal Hydraulic Conductivity

All of the cross sections were modeled as a multi-layered subsurface that extends from land surface at 9.5 feet NGVD to a depth of 100 feet below land surface. Horizontal hydraulic conductivities for each of the subsurface layers was estimated for two different scenarios in order to provide a range of seepage values for final design. One scenario (Scenario A) relied on horizontal hydraulic conductivity values determined from the geotechnical investigation, the APT, or effective hydraulic conductivity formulas, while the second scenario (Scenario B) relied on horizontal hydraulic conductivity values determined from the calibration of the 3-dimentional MODFLOW model of STA 3/4.

The geotechnical investigation was limited to the top 25 feet of subsurface material, which included three material types, peat, cap rock, and silty limestone. Samples of the peat were collected and hydraulic conductivity values were determined from laboratory falling head permeability tests (Nodarse Report – Section 3.3.2.3). The hydraulic conductivity of the limestone cap rock was determined from constant head permeability tests. The hydraulic conductivity of the silty limestone layer was determined from a combination of constant head tests performed on the entire 25 feet of subsurface material and an effective hydraulic conductivity formula combining the results of all three testing methods. The hydraulic conductivity of the lower 85 feet of subsurface material was determined from the results of a 96-hour APT test and an effective hydraulic conductivity formula.

4.3.4 Vertical Hydraulic Conductivity

The vertical component of hydraulic conductivity for each of the layers was estimated for the two scenarios in order to provide a range of seepage values for final design. Scenario A relied on published ratios of vertical to horizontal hydraulic conductivity, while

Scenario B relied on vertical hydraulic conductivity values determined from calibration of the 3-dimentional MODFLOW model of STA 3/4. Calibration of the MODFLOW model to the APT data resulted in vertical hydraulic conductivity values within the lower range of literature ratios of vertical to horizontal hydraulic conductivity. Because the APT test results are representative of only a small portion of the area of STA 3/4, it is not possible to determine if those values could be extrapolated to the entire STA 3/4. The concern was that the cap rock layers restricting vertical movement may not be continuous over the entire site or may be breached by drainage canals or other man-made features. Therefore, it was determined more prudent for design purposes to provide a range of seepage values based on both scenarios. **Table 4.4** summarizes the subsurface geology/lithology and the horizontal and vertical hydraulic conductivity for both scenarios in the vicinity of each of the cross sections. The vertical hydraulic conductivity for Scenario A is based on published values ranging from 20% to 50% of horizontal hydraulic conductivity, whereas, Scenario B is based on MODFLOW model calibration and values range from 0.5% to 5% of horizontal hydraulic conductivity.

4.3.5 Boundary Conditions and Stages

The vertical boundaries located at the ends of each cross section were chosen at distances where constant heads no longer influenced the groundwater seepage rate. The ends of the cross sections were modeled as open flow, constant head boundaries.

The perimeter levees, existing levees, seepage collection canals, supply canals, and existing canals were modeled with side slopes, depths, and widths based on conceptual design information provided by Burns and McDonnell. The vertical geologic profile was determined using information gathered from the geotechnical investigation of the STA and drill cuttings from the installation of the APT pumping and monitor wells.

Table 4.5 summarizes the water level stages used during the modeling runs. Three water level stages were chosen which correspond to the design maximum water levels (design), the average wet season, and the average dry season conditions. These water levels were

Insert Table 4.4

Table 4.5

STA 3/4 Stage Summary

Cross Section	Simulated Water Level		
	Design Maximum (ft NGVD)	Dry Season (ft NGVD)	Wet Season (ft NGVD)
East Perimeter			
STA 3/4	14.0	11.0	13.0
Seepage Collection Canal	9.5	9.5	9.5
North New River Canal	11.5	10.0	11.0
Farm Lands	7.5	7.5	7.5
North Perimeter			
STA 3/4	14.0	11.0	13.0
Inflow Canal	14.9	11.0	13.5
Seepage Collection Canal	7.5	7.5	7.5
Farm Lands	8.5	7.5	7.5
Supply Canal			
Supply Canal	16.3	11.0	14.0
Seepage Collection Canal	7.5	7.5	7.5
Farm Lands	8.5	7.5	7.5
Holey Lands	12.0	11.0	12.0
West Perimeter (North Section)			
STA 3/4	14.0	11.0	13.0
Holey Lands	12.0	11.0	12.0
South Perimeter			
STA 3/4	14.0	11.0	13.0
Holey Lands	12.0	11.0	12.0
West Perimeter (South Section)			
STA 3/4	14.0	11.0	13.0
Holey Lands	12.0	11.0	12.0
Outfall Canal	13.6	11.0	12.5

determined based on a range of expected conditions to estimate the seepage rates based on unit differences in head between the STA and the surrounding hydrology. These data are then input into the historical analysis discussed later in this report. The elevations in the surrounding area (farm lands) was based on the current operations which maintain the water table at approximately 18- to 24-inches below grade.

The SEEP2D model is a two-dimensional finite element model that offers a great deal of flexibility in developing the model grid. Resolution was increased in the area of the seepage collection canals and perimeter levees to provide a higher degree of accuracy. Reduced resolution was provided with increasing distance from the seepage canals. Maximum cell size was limited at the extreme ends of each model grid to 25 feet in length by 10 feet in thickness. **Appendix E** contains vertically exaggerated cross sections displaying the results of the seepage analysis. A cover sheet describing the cross sections is also included in **Appendix E**.

4.3.6 Seepage Analysis Results

Seepage values were determined for both scenarios (A and B) for the seven cross sections modeled for the STA. As discussed previously, the only difference between the scenarios is that Scenario B reflects the changes in hydraulic conductivity values obtained from the MODFLOW calibration and may be considered to represent the lower end of the seepage rates. Seepage rates were determined for the design maximum, wet season, and dry season water level stages. The results of the seepage analysis are presented in **Table 4.6A** and **Table 4.6B.** It was assumed that the lithologic layering is relatively consistent across the area associated with each cross section and remains consistent within the model boundaries.

4.3.7 Scenario A

The results of the seepage modeling for Scenario A (published ratios of vertical to horizontal hydraulic conductivity) indicate that supply canal losses range from a low of approximately 660,000 cubic feet per day per mile of levee (ft^3/d/mile) to a high of approximately 2,666,400 ft^3/d/mile. The percentage of seepage loss captured by the seepage collection canal ranges from 57 to 76 percent. Peak water levels in the farm lands adjacent to the supply canal exceed ground surface for the design maximum modeling run resulting in surface seepage. Surface seepage was also observed between the exterior levee and the seepage collection canal during the design maximum modeling run.

Along the northern perimeter and inflow canal, combined seepage losses from the inflow canal and STA range from a low of approximately 749,760 ft^3/d/mile to a high approximately of 1,467,840 ft^3/d/mile. The percentage of seepage loss captured by the seepage collection canal ranges from 77 to 83 percent. Peak water levels in the farm

Insert Table 4.6A and 4.6B

lands adjacent to the inflow canal exceed ground surface for the design maximum modeling run resulting in surface seepage. Surface seepage was also observed between the exterior levee and the seepage collection canal during the design maximum modeling run.

Along the eastern boundary of the STA, seepage losses from the STA range from a low of approximately 459,360 ft^3/d/mile to a high of approximately 1,562,880 ft^3/d/mile. Surface seepage was observed between the perimeter levee and the seepage collection canal in all modeling runs. Surface seepage was also observed between the seepage collection canal and the North New River Canal as well as east of the North New River Canal. The percent of seepage loss from the STA captured by the seepage collection canal range from 65 to 83 percent.

Along the western edge and the southern and western perimeters of the "Toe of the Boot", losses from the STA range from a low of approximately 142,560 ft^3/d/mile to a high of approximately 580,800 ft^3/d/mile. The seepage collection canal along the western edge of the "Toe of the Boot" also loses water to the Holey Lands at rates of approximately 21,120 to 211,200 ft^3/d/mile.

Review of **Table 4.6A** indicates that surface seepage occurs at the downstream toe of the exterior or perimeter levees under some scenarios, and could be detrimental to the stability of the levee system. Given that potential, **it is recommended that berms be constructed along the exterior or perimeter levees,** reducing the potential for surface seepage and concurrently affording the District means to more effectively maintain adjacent canals.

Groundwater mounding in the farm lands to the north ranged from 1.2 to 2.9 feet above the assigned "background" water table, depending on control water levels in adjacent canals. Is important to recognize that mounding in the water table most certainly exists today in the vicinity of the Holey Land and adjacent to the major Canals (North New River and Miami Canal) in the Everglades Agricultural Area. Although existing mounding has not been modeled as part of this effort, the fact that it does exist is an intuitive conclusion based on the stages maintained in these surface water features and the results of this evaluation.

Total seepage losses (water not captured by the seepage collection canal) from the STA, excluding losses from the supply canal and the southern most perimeter, were estimated based on the seepage rates in **Table 4.6A**. At the STA's maximum design depth condition, total seepage losses to regional flow were estimated at approximately 53,517 acre-feet per year (ac-ft/yr). During the dry season condition, total seepage losses were estimated at approximately 31,097 ac-ft/yr. Seepage losses along the eastern perimeter ranged from 6,615 to 10,169 ac-ft/yr, losses along the northern perimeter and inflow canal ranged from 9,225 to 13,295 ac-ft/yr, and losses along the western edge and southern and western perimeter of the "Toe of the Boot", to the Holey Lands, ranged from 15,257 to 30,053 ac-ft/yr.

4.3.8 Scenario B

The results of the seepage modeling for Scenario B (vertical hydraulic conductivity values determined from calibration of the 3-dimentional MODFLOW model of STA 3/4) indicate that supply canal losses range from a low of approximately 359,040 cubic feet per day per mile of levee (ft^3/d/mile) to a high of approximately 1,224,960 ft^3/d/mile. The percentage of seepage loss captured by the seepage collection canal ranges from 60 to 84 percent.

Along the northern perimeter and inflow canal, combined seepage losses from the inflow canal and STA range from a low of approximately 406,560 ft^3/d/mile to a high approximately of 739,200 ft^3/d/mile. The percentage of seepage loss captured by the seepage collection canal ranges from 38 to 46 percent. Peak water levels in the farm lands adjacent to the inflow canal exceed ground surface for the design maximum modeling run resulting in surface seepage.

Along the eastern boundary of the STA, seepage losses from the STA range from a low of approximately 285,120 ft^3/d/mile to a high of approximately 1,293,600 ft^3/d/mile. Surface seepage was observed between the perimeter levee and the seepage collection canal, between the seepage collection canal and the North New River Canal, and east of the North New River Canal in all modeling runs.. The percent of seepage loss from the STA captured by the seepage collection canal range from 28 to 80 percent. The seepage losses east of the North New River Canal were algebraically determined from the output of the seepage model runs. The values reported are for the water that leaves the STA. The path beyond the STA boundary is not calculated or evaluated. The retention area west of US27 has an invert elevation of 13 feet, which is above the water table. This drainage feature should have no affect on the seepage analysis.

Along the western edge and the southern and western perimeters of the "Toe of the Boot", losses form the STA range from a low of approximately 79,200 ft^3/d/mile to a high of approximately 232,320 ft^3/d/mile. The seepage collection canal along the western edge of the "Toe of the Boot" also looses water to the Holey Lands at rates of approximately 10,560 to 73,920 ft^3/d/mile.

Review of **Table 4.6B** indicates that surface seepage occurs at the downstream toe of the exterior or perimeter levees under some scenarios, and could be detrimental to the stability of the levee system. Given that potential, **it is recommended that berms be constructed along the exterior or perimeter levees,** reducing the potential for surface seepage and concurrently affording the District means to more effectively maintain adjacent canals.

Groundwater mounding in the farm lands to the north ranged from 0.8 to 2.1 feet above the assigned "background" water table depending on control water levels in adjacent canals. Is important to recognize that mounding in the water table most certainly exists today in the vicinity of the Holey Land and adjacent to the major Canals (North New River and Miami Canal) in the Everglades Agricultural Area. Although existing mounding has not been modeled as part of this effort, the fact that it does exist is an intuitive conclusion based on the stages maintained in these surface water features and the results of this evaluation.

Total seepage losses (water not captured by the seepage collection canal) from the STA, excluding losses from the supply canal and the southern most perimeter, were estimated based on the seepage rates in **Table 4.6B**. At the STA's maximum design condition, total seepage losses were estimated at approximately 44,021 acre-feet per year (ac-ft/yr). During the dry season condition, total seepage losses were estimated at approximately 28,553 ac-ft/yr. Seepage losses along the eastern perimeter ranged from 8,550 to 10,574 ac-ft/yr, losses along the northern perimeter and inflow canal ranged from 13,378 to 20,736 ac-ft/yr, and losses along the western edge and southern and western edge of the "Toe of the Boot", to the Holey Lands, ranged from 6,625 to 12,710 ac-ft/yr.

4.4 THREE-DIMENSIONAL SEEPAGE MODELING

A three-dimensional numerical groundwater flow model was developed as part of the hydrologic investigations and analyses of STA 3/4. The three-dimensional model was used to simulate the 96-hour aquifer performance test (APT) and provide an estimate of hydrogeologic parameters in the vicinity of the STA. Additionally, the three-dimensional model was used to calculate groundwater seepage and deep percolation losses from the STA.

4.4.1 Software Used

The modeling tool developed in this project was based on the U.S. Geological Survey's (USGS) three-dimensional groundwater flow model, MODFLOW (McDonald and Harbaugh, 1988). This model has been successfully applied to groundwater resource projects throughout southern Florida. To aid in the development of the MODFLOW input files and in the visualization of the model, the pre- and post-processor Groundwater Vistas by Environmental Simulations, Inc. was used.

4.4.2 Grid Discretization

The model area outside the immediate vicinity of STA 3/4 was discretized into 2,000 foot by 2,000 foot cells. Within STA 3/4, the model grid was variably-spaced. In the vicinity of the APT, the model grid spacing was between 25 feet and 200 feet. The small grid spacing in the vicinity of the APT matched the well spacing used to monitor water level changes. Throughout the rest of STA 3/4, the maximum grid spacing was 1,000 feet. The STA 3/4 groundwater model grid is shown in **Figure 4.7**.

4.4.3 Hydrogeology

The model area is underlain by the surficial aquifer system. The surficial aquifer system is an unconfined aquifer recharged directly by rainfall and surface water features. The geology and productivity of the surficial aquifer system is highly variable. In some intervals, it is comprised of transmissive sandy-limestones. Other non-productive intervals of the surficial aquifer system are comprised of clays, marls, hardpans, and less permeable sands.

4.4.4 Vertical Discretization

This model simulated the surficial aquifer and it was discretized into three model layers based on field testing results and the data provided in Fish (1988) (**Figure 4.8**) as follows:

- Layer 1 extended from the water table to approximately 15 feet below land surface (bls) and will contain the river, drain, and recharge cells. This layer generally consists of very hard limestone overlain by lower permeability muck and soil.
- Layer 2 extended from approximately 15 feet bls to approximately 25 feet bls, corresponding to the silty limestone overlain by the caprock at the site.
- Layer 3 extended from approximately 25 feet bls to approximately 120 feet bls. This layer corresponds to the transmissive zone at the site and consists

primarily of limestone. The bottom of this layer corresponds approximately to the top of the top of the lower transmissive zone in the surficial aquifer which becomes the intermediate confining unit at approximately 160 feet bls.

The hydrogeologic characteristics in each of these layers was homogeneous throughout the model grid. Initially, the hydrogeologic characteristics were estimated from the field testing program conducted as part of this project. The hydrogeologic characteristics were then refined as part of the calibration of the model to the APT.

4.4.5 Model Boundaries

The boundaries of the model were developed to minimize the interference on the model calculations in the immediate vicinity of STA 3/4. Boundaries to the model were simulated as constant heads. The constant heads were equal to the maintained water surface elevation in the land adjacent to the boundary. Constant heads were set equal to 9.5 feet south of the STA, adjacent to WCA 2A, and were set to 8.5 and 7.5 feet in the farmlands north of the STA.

4.4.6 Model Calibration

The 96-hour APT conducted on well TW-3 in STA 3/4 was simulated with the three-dimensional model to calibrate the hydrogeologic parameters used in the groundwater flow model (**Figure 4.9**). The APT was simulated as follows:

- Pumping – The cumulative flowrate from well TW-3 in ft^3/day was used as input to the well package. Pumping was estimated to be a constant 200 gallons per minute (gpm) for all stress periods.
- Recharge – The rainfall recorded during the APT was used as input to the recharge package. The rain gauge used to record precipitation during the APT was read seven times during the 96-hour test (**Table 4.7**). The rainfall during each record period was divided by the period length to estimate a uniform rainfall rate. Sherwood and others (1973) suggest that between 30 and 60 percent of rainfall becomes recharge to the surficial aquifer system in

Broward County. These values were used as initial estimates. The best fit to the APT occurred when 50 percent of the rainfall was modeled as recharge to layer 1 of the model. When these uniform recharge rates were used in the model, it became apparent that the rainfall recorded between August 6, 1999 at 3:10 PM and August 7, 1999 at 9:17 AM did not occur uniformly during the period. For the purposes of the model, it was estimated that 90-percent of the recorded rainfall occurred within the first hour of the recording period. The estimated rainfall is presented in **Table 4.7**.

- Canals – The east-west canals (north and south of the APT) were simulated in the calibration runs with the river package. These canals are shown in **Figure 4.9**.
- Time Discretization – Stress periods were developed to correspond with the rainfall data collected (**Table 4.7**).

Table 4.7

Rainfall Recorded During APT

Reading Time	Hours After Start of Test	Stress Period	Measured Rainfall (in)	Estimated Rainfall (in)	Estimated Rainfall Rate (ft/day)	Estimated Recharge Rate (ft/day)
8/4/99 15:14	2.1	1	0.00	0.00	0.0000	0.0000
8/5/99 9:30	20.3	2	0.09	0.09	0.0099	0.0049
8/5/99 15:05	25.9	3	0.03	0.03	0.0107	0.0054
8/6/99 9:46	44.6	4	0.03	0.03	0.0032	0.0016
8/6/99 15:10	50.0	5	0.00	0.00	0.0000	0.0000
8/6/99 16:10	51.0	6		0.31	0.6200	0.3100
8/7/99 9:17	68.1	7	0.34	0.03	0.0035	0.0018
8/8/99 12:45	96.0	8	0.04	0.04	0.0029	0.0014

4.4.7 Calibration Results

In order to determine the acceptability of the calibration model, the observed and modeled water levels at monitor wells 1d, 2d, 2s, 3d, 3s, and 4d were compared (**Figures 4.10 to 4.15**). The maximum difference between the observed and modeled water levels was less than 0.1 foot. Based on this comparison, the hydrogeologic parameters were determined to be sufficiently calibrated to the APT for determining seepage losses from the STA. The hydrogeologic parameters determined as part of the calibration are presented in **Table 4.8**. Additionally, the canal bottom sediment thickness and hydraulic conductivity used were 1 ft and 1 ft/day, respectively.

Table 4.8

Hydrogeologic Parameters in the Vicinity of STA 3/4

Layer	Horizontal Hydraulic Conductivity (ft/day)	Transmissivity (ft^2/day)	Vertical Hydraulic Conductivity (ft/day)	Storage Coefficient/ Specific Yield
1	100	1,500	0.5	0.1
2	200	2,000	0.38	2×10^{-5}
3	420	35,700	1.3	2×10^{-5}

The calibrated ratio of vertical to horizontal hydraulic conductivity was low for each of the layers (on the order of 1:200 to 1:526). This low ratio of vertical to horizontal hydraulic conductivity is likely due to interbedded thin sediment layers with a very low vertical hydraulic conductivity.

4.4.8 Comparison to SEEP2D Model

The hydrogeologic parameters developed during the calibration of the model to the APT were used to develop the MODFLOW seepage model. In order to verify that the seepage rates calculated by the MODFLOW model are comparable, they were checked against the seepage rates calculated by the two-dimensional seepage model (SEEP2D) for the lower vertical hydraulic conductivity values (described previously as Scenario B). The SEEP2D model provided an estimate of seepage lost from the STA and the supply canals and an estimate of seepage collected by the seepage collection canals for three design operating conditions. The seepage rates predicted by SEEP2D for four selected cross sections under the design condition were compared to the seepage rates predicted by the three-dimensional model. The results of these comparisons are presented in **Table 4.9**.

Table 4.9

Comparison of MODFLOW and SEEP2D Computed Seepage Rates

Flow Path	North Side		Cell 5 South		Cell 5 West		East Side (South)	
	Design Condition Seepage Rate (ft^3/day/mile)		Design Condition Seepage Rate (ft^3/day/mile)		Design Condition Seepage Rate (ft^3/day/mile)		Design Condition Seepage Rate (ft^3/day/mile)	
	MODFLOW	SEEP2D Scenario B	MODFLOW	SEEP2D Scenario B	MODFLOW	SEEP2D Scenario B	MODFLOW	SEEP2D Scenario B
From STA	128,832	310,992	134,640	233,376	133,584	228,096	293,568	906,576
To Seepage Collection Canal	349,008	337,920	--	--	--	--	296,208	661,584
From Supply Canal	219,648	427,680	--	--	--	--	--	--
To Regional Flow	81,312	392,832					176,352	681,120
To Holey Land	--	--	134,640	233,376	133,584	228,096	--	--
From North New River	--	--	--	--	--	--	121,440	604,560

Based on the comparison between the MODFLOW and SEEP2D models it was determined that the MODFLOW model with hydrogeologic parameters calibrated to the APT could be used to estimate seepage rates from the STA. The differences in seepage rates could be attributed to several factors including the following:

- Cross Section Interference. SEEP2D can not account for the effects that other cross sections may have on the seepage rates of the modeled cross section.
- Boundary Condition Interference. SEEP2D can not account for all of the effects of boundary influences in the vicinity of the STA, such as the Water Conservation Areas (WCAs) and agricultural drainage.

The MODFLOW and SEEP2D estimated seepage rates are similar, with the greatest differences occurring along the east perimeter. These greater differences could be due to the effects of the WCA located near this perimeter. The WCA is not modeled in the SEEP2D model. The model grid in the vicinity of the STA is presented in **Figure 4.16**. The modeled boundary conditions, including the Holey Land, WCAs, the supply canal, and agricultural drainage canals, are shown.

4.4.9 Model Runs

The three-dimensional seepage model developed from the calibration to the APT and the verification with the SEEP2D model was run for three design conditions: design maximum, wet season, and dry season. The stages associated with these scenarios were the same as those used in the SEEP2D model and are presented in **Table 4.10**. Supply Canal North and Supply Canal South refer to different segments of the supply canal. North Farm Lands are the farm areas north of the STA, but west of the North New River Canal. East Farm Lands are the farm areas east of the North New River Canal, north of WCA 3A.

Table 4.10

Stage Summary

Area	Simulated Water Level (ft NGVD)		
	Design	Wet Season	Dry Season
Supply Canal North	16.3	14.0	11.0
Supply Canal South	14.9	13.5	11.0
North Farm Lands	8.5	7.5	7.5
East Farm Lands	7.5	7.5	7.5
Seepage Collection Canals	7.5	7.5	7.5
STA 3/4	14.0	13.0	11.0
Holey Land	12.0	12.0	11.0
North New River Canal	11.5	11.0	10.0
L-5 Canal	11.5	11.0	10.0
Water Conservation Area 2	11.0	11.0	11.0

The results of the seepage analyses for these three runs are presented in **Table 4.11**. The results are divided into each of six perimeter segments corresponding to the perimeter segments used in the SEEP2D model. The East Perimeter South and East Perimeter North from the SEEP2D were combined and named “East Perimeter” for use in the MODFLOW model. The results from each of the three different design scenarios presented in **Table 4.11** vary because each represents a different water level scenario shown in **Table 4.10**.

Table 4.11

Seepage Results Summary

STA 3/4 Location	Seepage Rate (ft^3/day/mile)		
	Design Max	**Wet Season**	**Dry Season**
North Perimeter			
Loss From STA 3/4	109,824	108,240	79,728
Net Flow North of Supply Canal	310,464	265,584	160,512
Loss to Regional Flow	57,024	63,888	38,016
East Perimeter			
Loss From STA 3/4	213,840	167,904	77,616
Loss to Regional Flow	91,872	77,616	51,216
South Perimeter			
Loss From STA 3/4	125,136	98,736	46,992
Loss to Regional Flow	24,816	17,424	4,752
West Cell 3 Perimeter			
Loss From STA 3/4	84,480	33,264	(1,584)
South Cell 5 Perimeter			
Loss From STA 3/4	123,024	61,776	0.0
West Cell 5 Perimeter			
Loss From STA 3/4	128,304	62,304	(2,112)
Total Loss From STA 3/4 (ft^3/day)	3,483,645	2,590,783	1,181,697
Total Loss To Regional Flow (ft^3/day)	1,910,506	1,358,544	532,574

The seepage analyses show that the STA perimeters with seepage collection canals have less loss to regional flow compared to perimeters that do not have seepage collection canals. This shows that the seepage collection canals capture a significant amount of flow as expected.

In comparing flows between scenarios, losses to regional flow along the north perimeter are less in the design maximum scenario compared to the wet season and dry season scenarios. This is partially because the modeled stage in the farmlands to the north of this area are reduced in the wet season and dry season scenarios. The losses across perimeter sections with no seepage collection canals are much reduced in the wet season and dry season scenarios as compared to the design maximum scenario due to the lowering of water levels in the STA.

4.4.10 Sensitivity Analysis

To determine how changes in the model parameters determined during the model calibration could affect the seepage rates presented in **Table 4.11**, a sensitivity analysis was performed on the design maximum scenario run. The sensitivity analysis consisted of increasing and decreasing four parameters from those presented in **Table 4.8**.:

- STA/ Holey Land/ WCA River Package Conductance (increased 1000% and decreased 50%)
- Canal River Package and Canal Drain Package Conductance (increased 1000% and decreased 50%)
- Horizontal Hydraulic Conductivity (increased 50% and decreased 50%)
- Vertical Hydraulic Conductivity (increased 1000% and decreased 50%)

Two additional runs were made. The first consisted of using the maximum value for the parameters listed above and the second used the minimum value for the parameters listed above.

The results of this sensitivity analysis are presented in **Table 4.12**. The results indicate that the calculated seepage rates are most sensitive to both horizontal and vertical hydraulic conductivity. The model results are not as sensitive to the conductance values used in the MODFLOW river and drain package calculations. This helps establish the accuracy of the model. The conductance terms in the model are the least certain terms, but order of magnitude changes in the conductance only result in up to a 16-percent change in seepage from STA 3/4 and up to a 7-percent change in losses to regional flow. Meanwhile, the horizontal and vertical hydraulic conductivities were derived from calibration of the model to the APT and are similar to those derived by analytical APT analyses. The changes in horizontal hydraulic conductivity resulted in up to a 35-percent change in seepage from the STA and up to a 41-percent change in losses to regional flow. The changes in vertical hydraulic conductivity resulted in up to a 79-percent change in

Insert Table 4.12

seepage from the STA and up to a 46-percent change in losses to regional flow. The sensitivity analysis show that the model is least sensitive to the hydrogeologic parameters for which there is less certainty. This conclusion provides greater confidence in the modeling results.

The sensitivity analysis also shows that when the vertical hydraulic conductivity is reduced that seepage losses from the STA and losses to regional flow increase along the north perimeter (scenarios 8 vs. 7 and 10 vs. 9). This is likely due to the influence of the supply canal along the north perimeter. Lowering the vertical hydraulic conductivity reduces the ability of water from the supply canal to flow into the STA, thereby increasing the net flow from the STA.

4.5 CONCLUSIONS AND RECOMMENDATIONS

A hydrogeologic investigation has been performed at the site of the future Stormwater Treatment Area 3/4 (STA 3/4). This project included the installation of five test wells and six monitor wells along with a geotechnical evaluation (by Nodarse and Associates) to provide data for the preliminary design phase of STA 3/4.

This analysis was based on the proposed configuration of STA 3/4, as identified in the General Design Memorandum. Final cell numbering may change based on work completed after this task. However, internal modifications to the cells that do not effect stages (in the seepage collection system, STA 3/4 and Supply Canal) should not effect the results of this effort.

Data from the combined field effort (geotechnical and hydrogeological) were used in seepage analyses using two different computer modeling tools, SEEP2D and MODFLOW. These tools were used to estimate seepage losses from the STA 3/4 and the Supply Canal and provide preliminary values for use in the design of seepage collection systems surrounding STA 3/4.

Overall estimates of seepage (returned, lost, etc.) will be finally defined in the Plan Formulation Document in connection with the long-term hydrologic simulation (discussed in Part 7 of this document). The results of this analysis and the recommendations contained in this report will be used to further refine design parameters for the design of the project. The results of this analysis and the recommendations contained in this section will be subsequently used to:

- Define required maximum conveyance capacities for the seepage collection canals and seepage return pumping system(s). We recommend the use of the output from the SEEP2D Scenario A analyses for that purpose. Although the Scenario B output may result in a significant reduction in necessary pump and conveyance capacities, we feel that it would be prudent to incorporate the additional design capacity. The purpose behind this recommendation is to provide sufficient conveyance and pumping capacity to handle the higher flows predicted by the Scenario A SEEP2D runs. The ratios used in the Scenario A analysis are those used in the design of the seepage collection systems for other STAs and the Site 1 Pilot Reservoir. Prior to changing the approach to STA design, additional work may be warranted. This effort would require a comparison of actual seepage collection rates from operating STAs to the original design criteria to estimate the accuracy of the original predictions.
- Define overall seepage loss rates from and potential return to STA-3/4 in the conduct of a long-term hydrologic simulation (daily analysis over a 31-year period). Based on review of the data and the output, we believe the SEEP2D Scenario B results to be most appropriate for that use.
- Define varying water surface profiles in the seepage collection canals based on interior stages for use in establishing head differentials (which can vary along any given overall segment, such as the Supply Canal) to be employed in the hydrologic simulation. For this, we consider the SEEP2D Scenario B estimates to be most appropriate.

The recommended coefficients for use in development of the Plan Formulation Document are summarized in **Table 4.13**. These coefficients have been developed based on the data contained in this report but are expressed in terms on cubic feet per day of seepage flow per unit foot of levee per foot of head difference in stage between STA 3/4 (or Supply Canal) and the adjacent seepage canal. Only coefficients for the Design Condition are presented in this table.

Table 4.13

Recommended Seepage Loss Rates for Use in Design
(All values in cubic feet per day/foot of length/foot of head)

Source	**Range of Estimated Unit Losses**								
	Seepage Canal			**Holey Land**			**Net (Regional) Loss**		
	(1)	**(2)**	**(3)**	**(1)**	**(2)**	**(3)**	**(1)**	**(2)**	**(3)**
Supply Canal	N.A.	33.0	15.9	N.A.	34.7	7.1	N.A.	7.3	8.7
North Perimeter(4)	3.2	35.4	9.8	N.A.	N.A.	N.A.	1.7	7.2	11.4
East Perimeter, North(5)	7.4	44.7	14.6	N.A.	N.A.	N.A.	3.2	24.5	25.4
East Perimeter, South(5)	7.4	55.8	27.8	N.A.	N.A.	N.A.	3.2	28.7	11.3
West Perimeter	N.A.	N.A.	N.A.	12.2	54.8	22.1	N.A.	N.A.	N.A.
Cell 2A Outfall Canal(6)	N.A.	N.A.	N.A.	8.0	37.5	18.3	N.A.	N.A.	N.A.

(1) From MODFLOW analysis; use for long-term hydrologic simulations
(2) From SEEP2D Scenario A; use for maximum capacity of seepage collection canal and seepage return pumps.
(3) From SEEP2D Scenario B; use seepage canal hydraulic profiles from this estimate for computing head differentials to be used in long-term hydrologic simulations.
(4) Head differential between STA-3/4 interior and seepage collection canal stage.
(5) MODFLOW results for East Perimeter based on overall seepage
(6) Referred to as Cell 3 West in seepage modeling ("Toe of Boot"). Head differential between STA-3/4 interior and Holey Land stages.

The purpose of this investigation was to provide input data for the hydrologic investigation and design of the seepage collection system. The recommended coefficients discussed above are based on the results of this study along with experience gained on other STA projects in the Everglades Agricultural Area. Coefficients for the design of the seepage conveyance system are based on the more conservative values predicting higher seepage rates to provide sufficient capacity in the system to capture the seepage; the resultant seepage collection system analysis is presented in Section 8 of this Plan Formulation document. The recommendations for the hydrologic simulation (presented in Section 7 of this Plan Formulation document), which estimates overall seepage volumes computed over a 31-year period, include the less conservative values developed from the MODFLOW modeling of the APT.

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